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# Unusual Behaviour of an Earth-Rockfill Dam

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SYNOPSIS: The post-construction behaviour of a 47.5 m high and 700 m long earth-rockfill dam, located in northwestern Iran, is described. The embankment dam is founded on Tertiary rock on both abutments, but in the river channel it rests on alluvial deposits of variable composition and thickness. Since the first impoundment, the dam has undergone large deformations, both in vertical and in horizontal directions. These have resulted in unusual differential settlements, visible along the crest, and have produced wide longitudinal cracks in the asphalt pavement of the crest road. The principle features of the dam are described and selected monitoring results of the crest movements for a sixteen year period are presented. Based on the analysis of the available data and taking into account the geotechnical characteristics of the foundation and the fill materials, an interpretation of the unusual behaviour has been attempted.

## INTRODUCTION

Behaviour control of large fill dams is of great importance to both the dam (geotechnical) engineer and the dam owner. For the dam engineer every large dam usually constitutes a unique experience, because dam geometry, subsurface conditions, construction materials, and construction procedures are never the same, and continuous observation of the behaviour of a constructed facility will show whether the design was successful. For the dam owner, on the other hand, safety, efficient operation and serviceability are the main concerns. Past experience has shown that good site investigation practice and adequate instrumentation are key factors in successfully predicting the future behaviour of a dam. Prediction in turn, forms the basis for implementing appropriate measures during construction to guarantee a satisfactory performance during the dam's service life.

In this paper, the unusual deformation behaviour of Mahabad dam will be presented. After first describing the principal features of the dam and its foundation, selected data on the present condition of the structure and its performance during the past 16 years of operation will be given. Finally, an attempt will be made to interprete the data in the light of subsurface conditions and properties of the fill materials and to give possible causes for the deformations observed.

## PRINCIPAL PROJECT DATA

The Mahabad earth-rockfill dam, located in northwestern Iran, forms part of a multipurpose project which provides irrigation for some 20,000 ha of agricultural land. Furthermore, it generates power from a small plant for Mahabad Town and the villages in the irrigation area, and finally, it stores potable water for Mahabad Town. The dam was completed in 1970 and filling of the reservoir started subsequently. A layout of the facility is shown in Fig. 1.

### Embankment Dam and Reservoir

The dam has a maximum height of 47.5 m, a slightly curved, 700 m long crest, and a base width at the valley floor of 221.5 m. A typical cross-section is displayed in Fig. 2. The fill volume is about 1.5 million $m^3$. The maximum normal operating water level of the reservoir is at 1358.5 and the minimum at 1333.0. This gives a useful storage of 180 million $m^3$.

The dam has a rather narrow central clay core with a slight inclination in the lower 2/3 of its height. The width at the top is 4.0 m and at the bottom 11.0 m. The other material zones can be seen from Fig. 2. The surface of the D/S (downstream) face is covered by a placed top soil layer with planted grass.

### Engineering Geology and Foundation Conditions

Lithostratigraphy. - Basically, the dam site consists of Cretaceous rocks underlying Neogene deposits which in turn are overlain by Quaternary deposits. The original width of the river plain was about 250 m. Figure 3 shows a cross-section through the valley and dam abutments along the dam axis.

On the left abutment the dam rests first on Cretaceous bedrock for some 50 m. The Cretaceous series consists mainly of shales and slates which are intercalated by limestone beds. For the following 150 m the foundation consists of Neogene rocks of variable thickness and compressibility, but supposedly well consolidated. These are conglomerates and siltstones, but in the only borehole drilled in this section, they are described as "silty clay with lenses of sandy clay and some

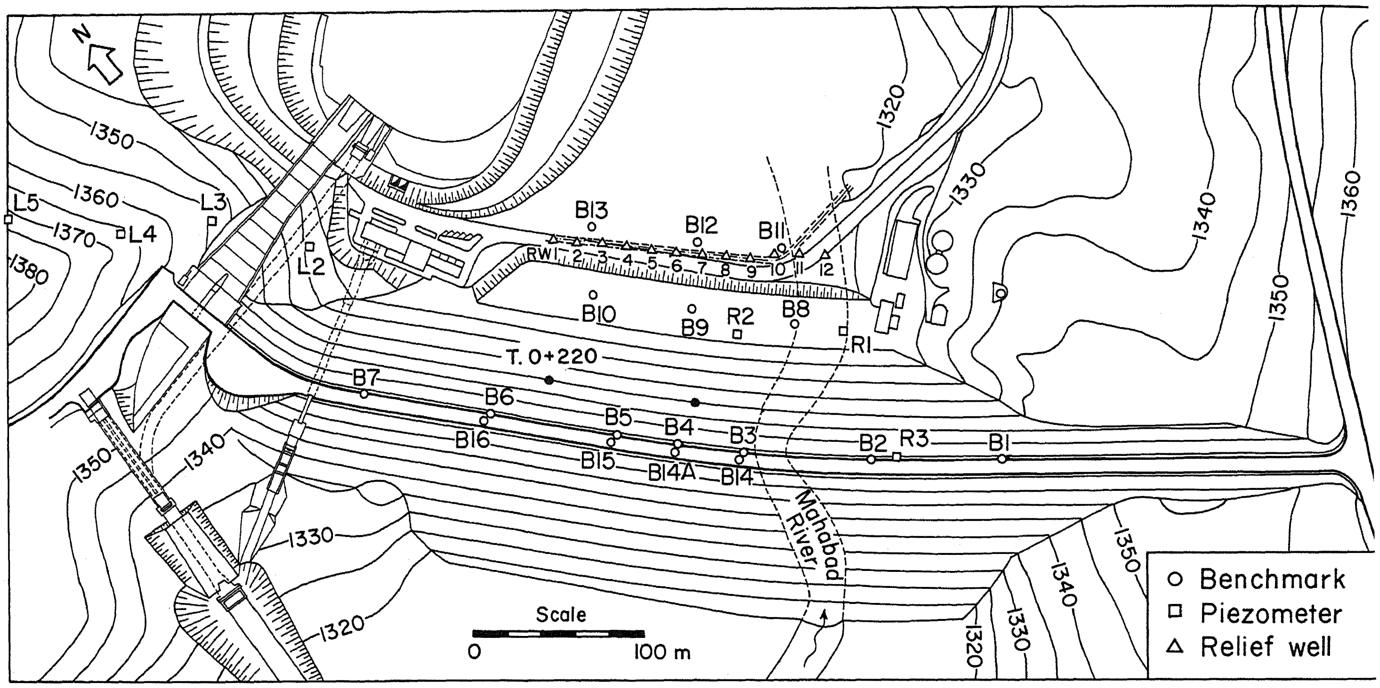



Fig. 1 Layout of Mahabad dam showing locations of available monitoring facilities

pebbles" reaching a depth of 29 m.

In the river channel, the Cretaceous formation occurs at a depth of 45 to 48 m. It is overlain by Neogene, Pleistocene and Holocene deposits. The Neogene consists of siltstone, silty sand with lenses of gravel and pebbles, and the thickness of the beds varies from about 25 m on the left side to 8 m on the right side of the valley. The Pleistocene beds are composed of silty clay, sand and silty gravel. The Holocene at the top has an average thickness of about 6 m and contains predominantly silt, sand and gravel. The alluvial fill is very heterogeneous and does not show a continuous stratification.

On the right abutment the supporting rock consists of a thick body of travertine which has formed from (still active) mineral springs between chainage 0+440 and 0+610. It lies partly on the weathered surface of the Cretaceous shale and slates and partly on a thin, lenticularly-shaped remnant of Neogene rock. Further up the right abutment there are again Neogene deposits in the form of clay, silty clay, sand and gravel at the bottom and breccia at the top. They are overlain by a thin mantle of Quaternary silty clay and sand.

Properties of foundation materials. - During the final design phase, the subsurface was inve-

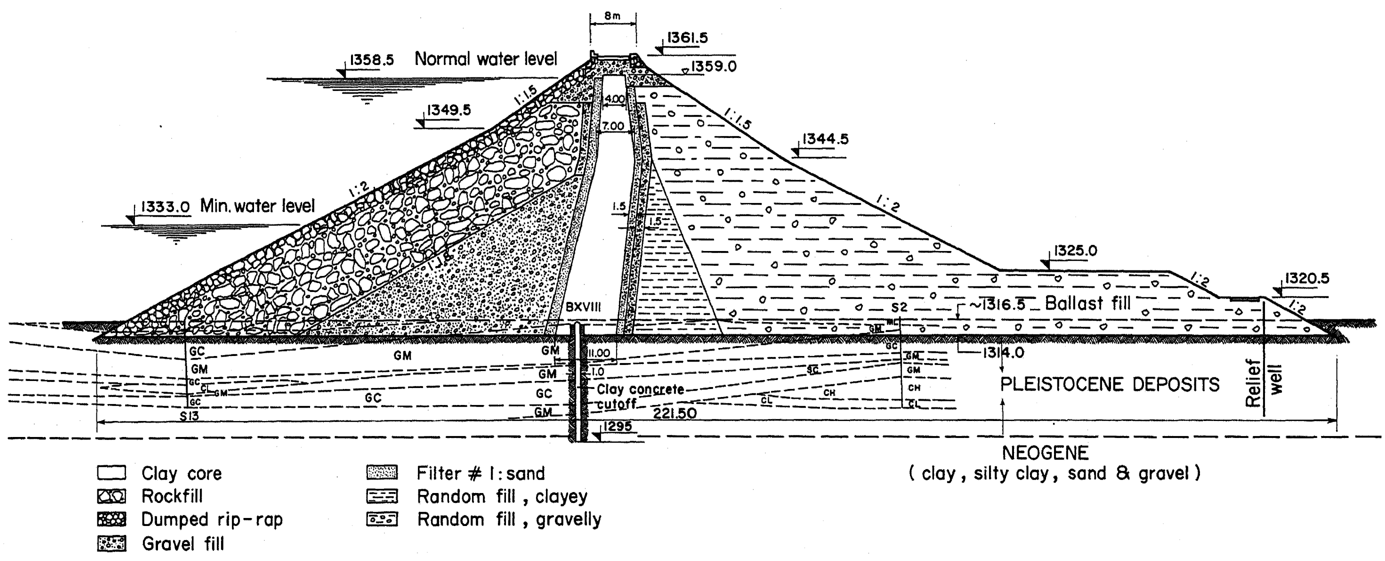



Fig. 2 Cross-section of dam showing material zones

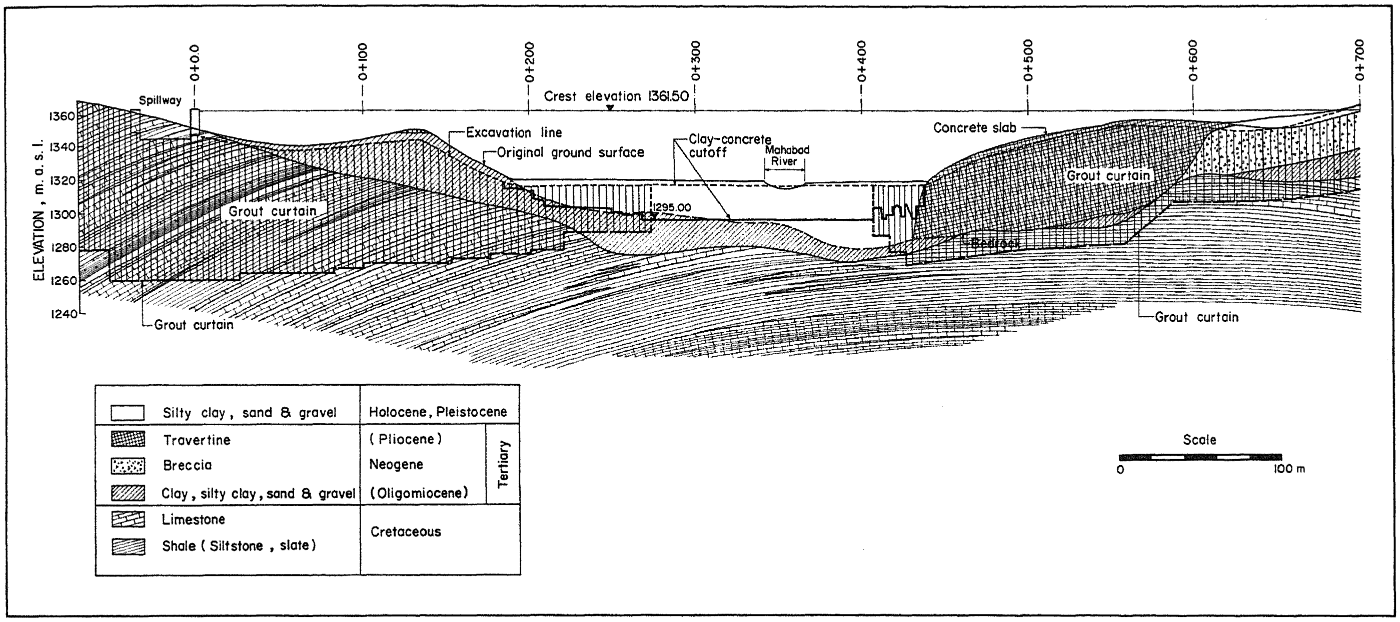



Fig. 3 Longitudinal section along dam axis (looking D/S) showing geology, grout curtain and cutoff

stigated by more than 50 boreholes. Most of these were drilled to shallow depths (15-30 m), but a few also reached down as far as 100 m. The logs revealed the very erratic character of the subsoil. Silty sands, clayey and silty gravel and low-plastic clays were abundant; high-plastic clays, on the other hand, were less common.

In many boreholes standard penetration tests (SPT) were conducted and typical blow counts in the upper 15 m of the alluvium were around 30 with minimum values of about 15. Blow counts varied irregularly and increased as well as decreased with depth in the same borehole.

Permeability tests revealed that there were surprisingly many zones with a k-value of less than $10^{-6}$ cm/s or between $10^{-3}$ and $10^{-6}$ cm/s. This indicated that the subsoil may actually have been finer-grained than inferred from the boring logs. It is possible that with the drilling method employed some of the fine material had been washed out.

Except for grain size analyses, only few laboratory tests were carried out. Of interest here are oedometer tests on CL and CH soils. For CL soils, the compression index, $C_c$, of samples taken from as deep as 40 m typically varied from 0.17 to 0.40 ($e_o$ = 0.40-0.75) when the load increased from 0.8 to 1.6 MPa. CH soils gave higher $C_c$-values, e.g. in a borehole 45 m D/S of the dam axis at about chainage 0+250, a sample from 12 m depth had $C_c$-values of 0.65 to 0.70 ($e_o$ = 1.45).

## Seepage Control

Means employed for the control of seepage and to prevent piping were a grout curtain in the abutments and a continuous clay-concrete diaphragm wall along the central part of the dam (see Fig.3). The diaphragm wall was 20 m deep and 3 m wide. It cut off fairly pervious Holocene deposits and penetrated into less permeable Pleistocene layers. In the abutments the cutoff was continued as shallow trench excavated through the weathered bedrock. In travertine, the excavation was about 3 m deep and the bottom of the trench was lined with a concrete slab.

An additional control feature which was thought necessary was a row of twelve relief (bleeder) wells, spaced 15 m apart, which are located at the edge of the D/S berm (see Figs. 1 & 2). The top of the relief well is at El. 1320.5. The water level in these wells, however, always remains below the ground elevation.

## Embankment Fill Materials

Engineering properties of the embankment fill materials are summarized in Table I.

The core material was compacted with a Dynapac CF 44 vibrating sheepsfoot roller in lifts of 30 cm with 6 passes. Field compaction results showed that the field compaction curve was between the standard Proctor and the modified Proctor curves and test results plotted mainly on the wet side with respect to the field compaction curve. Frequency curves of compaction test results demonstrated that 100% of all dry density values were above 95% of the standard Proctor maximum dry density and about 70% were higher than 100% Proctor standard. The most frequent values were between 16.4 and 16.6 kN/m$^3$. About 60% of the tests were drier than the standard Proctor optimum water content. The fill water contents varied between about ±3% with respect to the optimum value which was around 20%.

The random fill and the ballast fill were spread in layers of 30 to 50 cm and compaction was effected by two passes of a 5.4 t vibratory roller.

TABLE I Material properties

| Material type | Granulometry $D_{15}$ $D_{85}$ (mm) | $\gamma_d$ (kN/m$^3$) | c' (kPa) | $\phi$' (°) | k (cm/s) |
|---|---|---|---|---|---|
| Core | 0.004<br>0.025-0.2 | 15.8-17.0 | 70-140 | 27-30 | $5 \cdot 10^{-8}$ |
| Filter I | 0.02- 0.1<br>0.5 - 2.0 | 14.7-16.7 | - | - | $10^{-4}$ |
| Filter II | 0.4 - 2.0<br>15 - 40 | 17.7-18.6 | - | - | - |
| Random | | 18.6-19.6 | - | - | - |
| Rockfill | > 8<br>> 127 | 18.6-19.6 | - | 42 | - |
| Gravel fill | | 19.6-20.6 | - | - | $10^{-3}$ |

Gravel fill was densified in layers of 50 cm with two passes of a 4.6 t vibratory roller. A relative density of 70-75% could be achieved.

The rockfill of the U/S (upstream) shell was placed in lifts of 1.5 m and compacted with four passes of a 15 t vibratory roller. Bulk densities of 19.6 to 21.1 kN/m$^3$ could be achieved.

Finally, the rip-rap was placed in its full-course thickness.

Construction History of the Embankment

Construction of the embankment started in June 1968 with fill placement to the left of chainage 0+340 (which was the edge of the river). It continued until October and reached El. 1346. The slope of the fill towards the river (in the direction of the dam axis) was rather steep, probably as much as 1V:1H. Filling was resumed in June 1969 when the river had been diverted through the bottom outlet tunnel. First the breach on the right hand side of the valley was filled and then placement proceeded over the entire length of the embankment. The final elevation was reached in December 1969.

Instrumentation and Monitoring Facilities

Facilities to assess embankment performance, both during construction and after putting the dam into service, have been limited to the observation of surface movements and water levels. The following devices and installations are presently available (see Figs. 1 & 4):

(1) for deformation measurements:

- Seven target points (or benchmarks) B1 to B7, located on the D/S curbstone of the crest between chainage 0+100 (B7) and 0+485 (B1).

- Three benchmarks, B8, B9 & B10 on the D/S

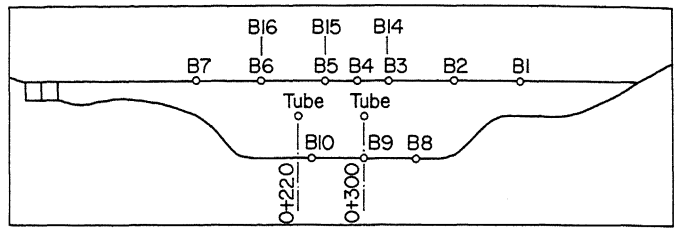



Fig. 4 Longitudinal section showing locations of benchmarks and settlement tubes

berm of the embankment.

- Three benchmarks B11, B12 & B13 located 15m D/S of the foot of the berm.

All these benchmarks could be used to monitor both vertical and horizontal displacements (with azimuth).

- Four benchmark shafts B14, B14A, B15 & B16 along the dam axis on the crest to monitor the vertical and horizontal movements of the top of the clay core. They consist of a 0.4 m ID steel pipe inside a concrete-lined shaft placed on the clay core. Inside the pipe there is a Ø 35 mm steel bar, about 1.5 m long, pushed into the clay such that its top, which is used as a reference, is 0.5 m above the top of the core (installed at the end of 1973).

- Two pipes with a steel plate at the foundation level (approx. El. 1314), 31 m D/S from the dam axis at chainages 0+220 and 0+300. Vertical movements are measured at the top of the pipe (installed in June 1968).

- Twenty-four points (marked by paint) on the U/S curbstone and seventeen points on the D/S curbstone (in addition to the benchmarks) for the measurement of vertical displacements (settlement).

(2) for the measurement of pore water and uplift pressures:

- Three open standpipe piezometers R1, R2 and R3 in the D/S foundation of the embankment on the right side of the valley, and four piezometers in the left abutment, i.e L2 to L5.

- Twelve relief wells, RW1 to RW12, located along the edge of the D/S berm (see Fig. 1).

(3) for the measurement of seepage water or leakage through the dam:

- a grouting gallery in the right abutment

- a ditch with V-notch weir located at the foot of the dam on the right side of the valley (near to the water treatment plant).

Data from piezometers and relief wells, as well as from seepage, will not be discussed here; the limited records do not indicate unusual behaviour.

PRESENT CONDITION AND AVAILABLE DATA ON DEFORMATION BEHAVIOUR

Dam Crest

The constructed width of the crest was 8.32 m and consisted of a 6 m wide asphalt road and concrete curbstones, 0.30 m high, on both sides. On the D/S side the 0.82 m wide curbstone is supported by dry rubble fill which slopes with 0.8H:1V. It has provision for seating lamp posts and also contains the geodetic target points. The 1.50 m wide U/S curbstone is seated on about 1 m of dry rubble fill with a similar slope as on the D/S side. It contains 0.7 m high masonry wall segments connected by a continuous guard rail. The crest road had an initial camber of 1.0 m.

The present condition of the crest clearly shows that the dam has undergone a complex pattern of differential movements. Figure 5 shows a map of the crest with some of the more prominent features of distress. The most striking ones are the wavy edge of the D/S curbstone, longitudinal cracks in the asphalt pavement, and a tilting of the crest towards U/S. In the following some more details on the appearance of the crest are given:

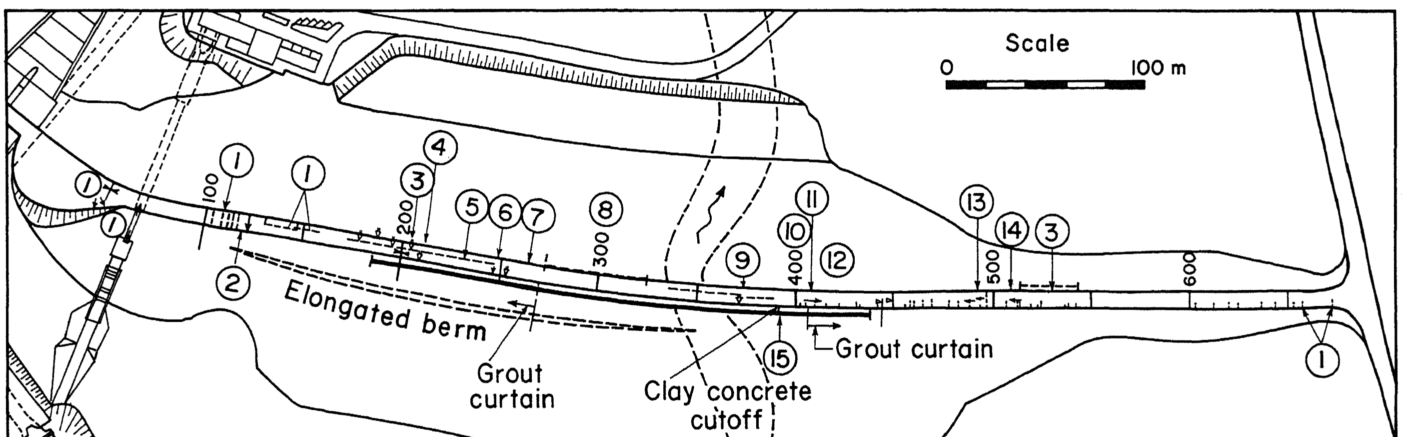
- Transversal cracks appear around chainage 0+110 and also from 0+450 to about 0+500. These cracks occur because of differential settlements between the abutments and the valley part and are a common phenomenon for dams on compressible foundations (Blinde, 1987).

- Longitudinal cracks appeared between chainage 0+120 and 0+250 and again between 0+335 and 0+370. (A crack opened for the first time close to the D/S curbstone on May 1, 1972, when the reservoir level rose to El. 1357.5 for the first time). Some of the cracks had widths of up to 15 cm, and some also showed a vertical offset of the asphalt by as much as 10 cm, whereby the U/S part settled with respect to the D/S part. The cracks were later filled in with sand and mortar and most of them have remained closed since.

- Between chainage 0+265 and 0+325 the D/S curbstone has buckled forming an arch over the underlying rubble fill with an uplift of about 30 cm.

- At about chainage 0+400 compressive forces have sheared the U/S curbstone and guard rail and produced an overlap of the sheared parts.

- There is a considerable tilt of the road pavement towards U/S. The positions of the levelling points on the D/S and U/S curbstones are shown in Fig. 6, together with the original elevation, for June 1986. The largest differential settlement (tilt) between the two curbstones occurs at 0+226, and on June 14, 1986 it had reached 64 cm. (To get the pavement elevation, the height of the curbstone, usually 30 cm, must be subtracted). The camber at this




Legend :

1 Cracks
2 U/S side of crest with higher settlement
3 Depression
4 Rel. peak in curbstone elevation
5 Cracks filled with concrete
6 Wavy pavement
7 Rel. peak
8 30 cm arching of curbstone
9 Pavement significantly inclined
10 Visually deepest point
11 BM plate tilted
12 Crest heave
13 Visible settlement starts
14 Highest point
15 Curbstone and guard rail sheared

Fig. 5 Crest map showing some of the distress features

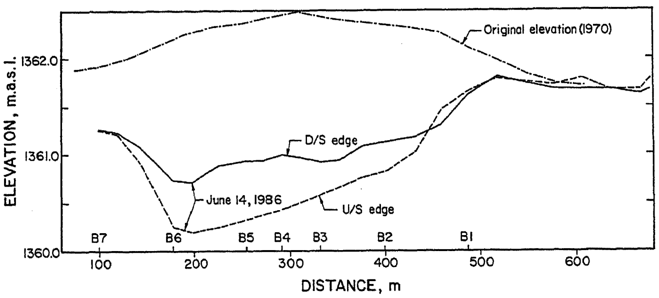



Fig. 6 Differential settlements of U/S and D/S curbstones along dam crest

section was originally 0.85 m and hence the settlement at the centerline of the pavement amounts now to more than 2 m.

## Vertical Displacements

The time variation of vertical displacements, i.e. the vertical components of the displacement vectors, of crest benchmarks B3, B5, B6 & B7 and of core benchmarks B14, B15 & B16 are shown in Fig. 7. Also shown are the rates of settlement of the ground surface plates (tubes) at 0+220 and 0+300. The positions of these curves are, however, arbitrary. The value of the total settlement (since June 1968) is obtained by adding 700 mm to the settlement value shown by tube 0+220 and 950 mm to that of tube 0+300. The reliability of these ground settlements is, however, questionable, because there is no sleeve provided between the settlement pipe and the fill material.

During the first phase of fill placement to El. 1346, the settlements of tubes 0+220 and 0+300 were 520 and 830 mm respectively. The corresponding stress increase was approximately 0.55 MPa. During the following eight months of no fill placement, the settlements increased by 90 and 120 mm respectively. When fill placement was completed in December 1969, additional settlements of 130 and 340 mm respectively had taken place with an estimated stress increase of 0.12 MPa. From these data the constrained modulus of the foundation can roughly be estimated as 10 to 20 MPa (assuming a compressible layer thickness of about 20 m).

For the vertical movements the first four filling and drawdown cycles are of particular interest. During the first year of impounding (1971) the reservoir level reached a peak of 1351. Settlement rates started to increase when the level had reached 1342.5. They decreased soon after the peak had been exceeded. During the second reservoir peak in 1972, settlement rates started to increase when the reservoir level reached El.1353, and decreased again when it receded to El. 1355. Interestingly, an increase also took place during the very deep drawdown in winter 1972/73. Benchmarks B8, B9 & B10, on the other hand, showed a slight rebound of about 2 cm. No increase in the rates of settlement was observed during the following reservoir peak in spring 1973 which attained only El. 1353. A further increase occurred in summer 1974, especially also with the newly installed core benchmarks B14 to B16. It started when the reservoir level reached about El. 1345. Another increase in settlement rates can be noticed in 1976, but since then settlement rates have decreased steadily without showing a strong relationship with reservoir level fluctuations.

Settlement rates at benchmarks B8 to B13 remained quite small and were not affected by the reservoir level. Tubes 0+220 and 0+300 had a much reduced settlement rate when compared with B3 to B6

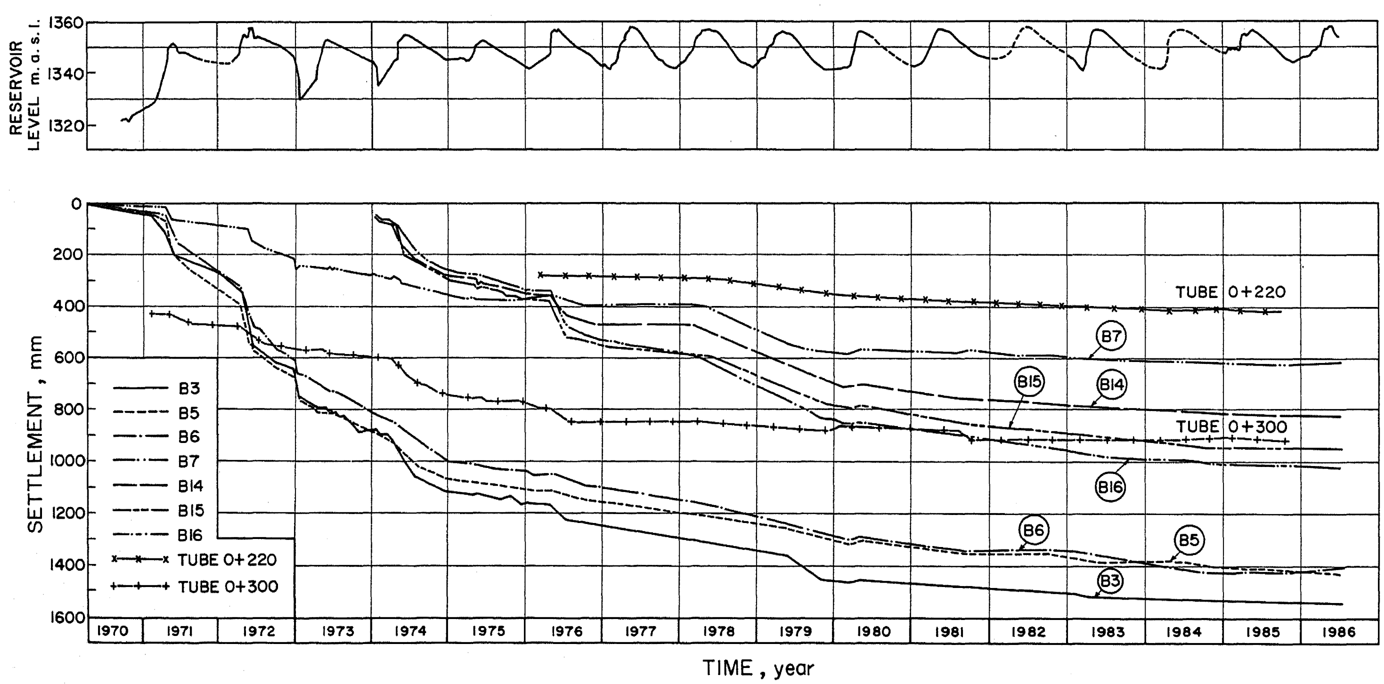



Fig. 7 Vertical components of displacement vectors (settlement) of selected points

and were not very sensitive to the water load imposed.

The rates of settlement of the top of the clay core, as observed by the shaft benchmarks B14 to B16, were higher than those of the corresponding curbstone benchmarks. For example, since installation at the end of 1973, B16 has settled 1020 mm while B6 has settled only 610 mm. This is because of the rigidity of the curbstone which may cause arching.

The distribution of the vertical displacements between B1 and B7 is more or less symmetric with respect to B4, i.e. the center of the valley.

Horizontal Displacements

Figure 8 shows the time variation of horizontal displacements (i.e. the components of the displacement vectors perpendicular to the longitudinal axis of the dam) of benchmarks B3, B5, B6, B7 and of B14, B15 & B16. It can be seen that during the first filling cycle in 1971, the crest points moved first towards U/S. Only after the peak had been exceeded, i.e. about 4 months after start of impounding, the movements reverted to the D/S direction. Benchmarks on the D/S berm, B8 to B10, on the other hand, always moved towards D/S, but at a much smaller rate.

The time variations of the horizontal displacements are similar to those of the vertical components (settlement), i.e. accelerations in the movements can be noticed when the reservoir level reached 1353 during the 2nd filling in 1972 and again when it reached 1345 during the 4th cycle in 1974. During the deep drawdown in winter 1972/73 there was a small rebound and the crest moved U/S by about 3 cm.

In a horizontal plane the displacement vectors point to the center of the valley, i.e. B1 to B4 move to the left, while B5 to B7 follow a path turning to the right. The displacement pattern with respect to the valley is not symmetric; the largest horizontal displacement has occurred at B6.

## DISCUSSION

Large settlements of dams on compressible foundations are as such not unusual. In the case history presented here, the unusual behaviour refers to the underestimation of the settlements and the relatively large horizontal displacements and tilting of the crest.

An analysis of the deformations Mahabad dam has

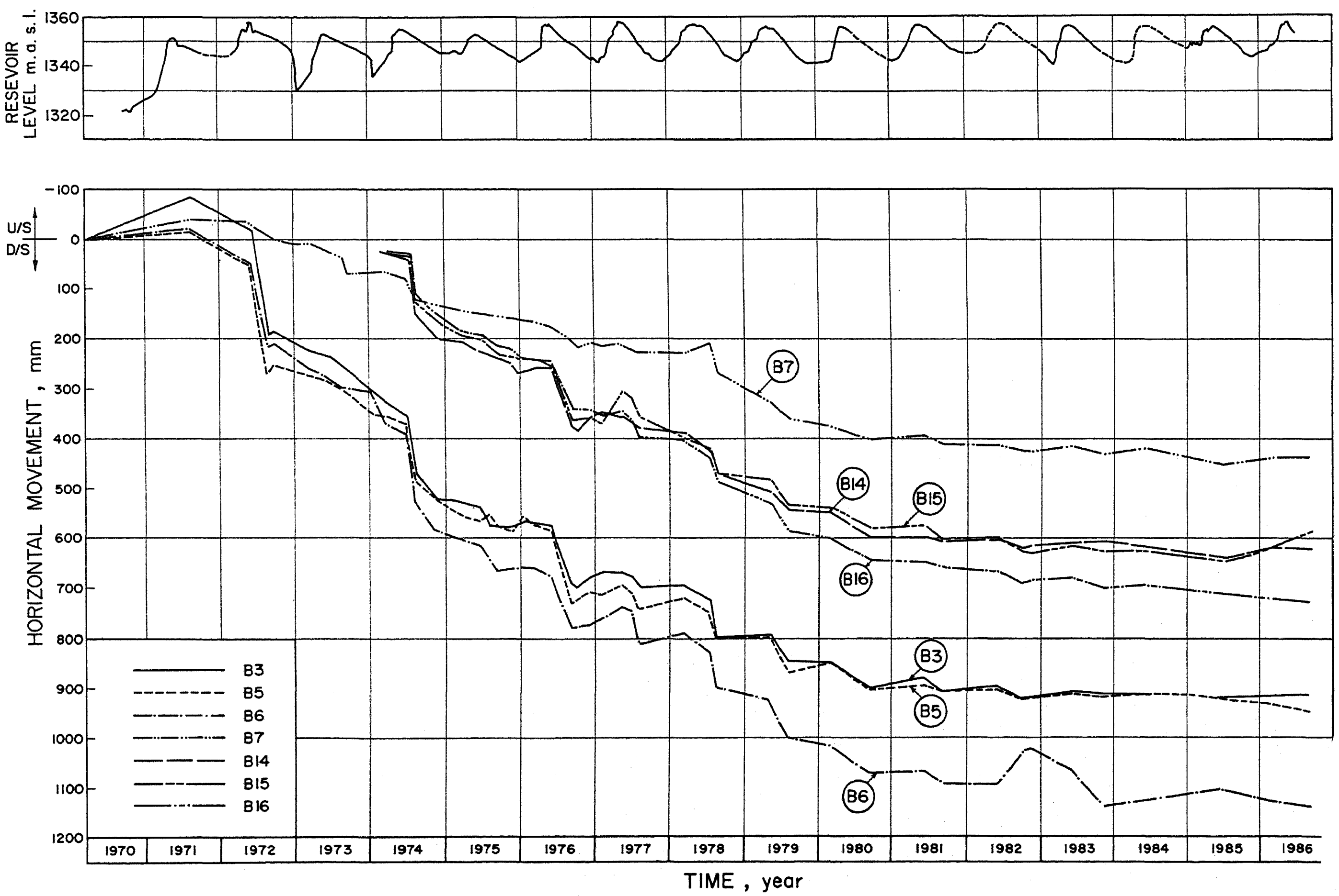



Fig. 8 Horizontal components of displacement vectors of selected benchmarks

experienced must involve some speculations because the data now available are not sufficient to find out the true mechanism with certainty. The main factors which have most likely contributed to the unpredicted differential movements are: (1) a foundation which was more compressible than anticipated from site investigation, and (2) a strength decrease (weakening) with time of the U/S rockfill under submerged conditions or under wetting and drying cycles. In the following, a possible mechanism is put forward which would explain the phenomena observed.

During the first filling, the U/S shell settled partly because of the wetting of the rockfill, which caused a strength decrease, and partly from the compression of the relatively impervious lake bottom U/S of the dam axis, which was wetted for the first time, under the water load. This produced a tilt of the crest towards U/S with a horizontal displacement component in the same direction. The importance of the compressible foundation in contributing to this mechanism has actually been verified by physical model and finite element studies (Alberro et al, 1976). During the 2nd filling, the water load was considerably larger than in the previous year (it increases with the square of the depth of the impounded water) and the rather plastic core was pushed on the D/S shell causing the crest to move in D/S direction (compare also with the analysis given by Nobari & Duncan, 1972). In addition, the steeper U/S part of the U/S shell became submerged for the first time; its settlement produced additional tilting. The settlement of the U/S shell in combination with the D/S movement of the crest produced longitudinal cracks which became visible in the rigid asphalt pavement. These cracks were probably also associated with a sliding movement. The D/S curbstone which originally had not been divided into segments buckled as a result of the compressive forces developed by the D/S movement. The settlement of the D/S shell remained small, as evidenced from benchmarks B8 to B10 and also from the foundation settlement plates (e.g. tube 0+300).

When the dam was subjected to a deep drawdown in winter 1972/73, the U/S shell, for the first time after about 1 1/2 years of submergence, experienced an increase in effective stress leading to additional settlements. It must also be assumed here that parts of the rock in the rockfill have been weakened during soaking. This hypothesis can be substantiated by the facts that also in the rip-rap (for which rock of the highest available quality was employed) some large blocks show now a splitting up along numerous fissures. In the quarry from which the rockfill had been obtained, the rock is often highly fissured and in some places also platy. Tests on these rocks also showed that after wetting the compressive strength was reduced to one half or even less.

It is believed that most of the settlement observed after the dam was completed was due to the weak foundation which was more clayey than originally inferred from site investigation and indicated in the foundation of Fig. 2. The dam was well-built according to the monthly construction reports and usually a dam of this height does not settle more than about 20 cm if it were founded on rock (max. 0.3% of its height according to Dascal, 1987). The rate of settlement and also that of horizontal movement have steadily decreased and it can be concluded that primary consolidation has now been completed. Further settlements will be due to creep.

## CONCLUSIONS

The case history presented here demonstrates first the importance of instrumentation in large dams. Had the dam been properly instrumented, especially with respect to settlements and lateral movements, in both the foundation and the dam body, it would have been possible to predict future settlements after completion of construction fairly accurately and an appropriate camber could have been provided. It also would have been possible to clearly separate the settlements taking place in the foundation from those occurring in the dam body.

Second, the benefit of a good quality site investigation which is able to identify the true characteristics of the subsurface strata, must be emphasized. Layers with low permeabilities, even if coarse materials appear to make up the bulk, will not complete their consolidation during construction and will be a source of long-term movements.

## ACKNOWLEDGEMENTS

The work presented herein forms part of an ongoing study on the monitoring facilities and safety of Iranian dams carried out by Mahab-Ghodss Consulting Engineers, Tehran, in association with the Joint Venture Stucky-Electrowatt, Lausanne/Zürich. Much of the data given here has been compiled from numerous reports and records stored in the archives at the dam site and the provincial water board. The cooperation of the staff of Mahabad dam site in making these sources of information available is gratefully acknowledged.

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